AD-A106 638   BLACK AND VEATCH KANSAS CITY MO   F/G 13/13
NATIONAL DAM SAFETY PROGRAM. INTERNATIONAL AIRPORT DAM (MO 1066--ETC(U)
APR 79 P R ZAMAN, P B MACROBERTS   DACW43-79-C-0040

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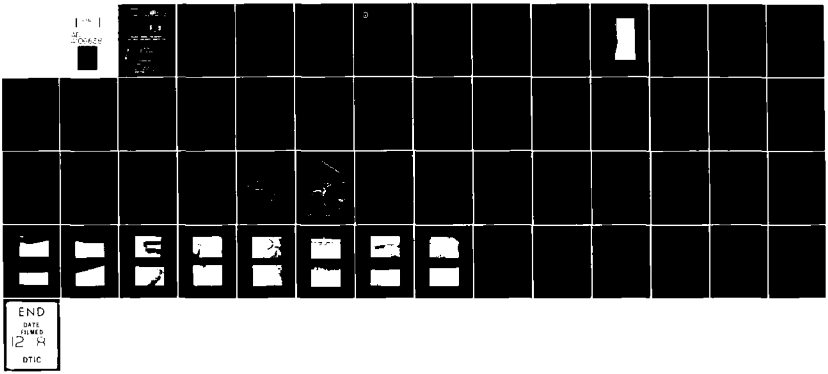

LEVEL II

MISSOURI-KANSAS CITY BASIN

AD A106638

INTERNATIONAL AIRPORT DAM

PLATTE COUNTY, MISSOURI

MO 10661

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY INSPECTION

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

PREPARED BY: U.S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

APRIL 1979

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REPORT DOCUMENTATION PAGE

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1. REPORT NUMBER — 2. GOVT ACCESSION NO. AD-A106	3. RECIPIENT'S CATALOG NUMBER 638
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9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Engineer District, St. Louis Dam Inventory and Inspection Section, LMSED-PD 210 Tucker Blvd., North, St. Louis, Mo. 63101	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
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National Dam Safety Program. International Airport Dam (MO 10661), Missouri - Kansas City Basin, Plate County, Missouri. Phase I Inspection Report.

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dam Safety, Lake, Dam Inspection, Private Dams

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.

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MISSOURI-KANSAS CITY BASIN

INTERNATIONAL AIRPORT DAM
PLATTE COUNTY, MISSOURI
MO 10661

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PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY INSPECTION

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

PREPARED BY: U.S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

APRIL 1979

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 NORTH 12TH STREET
ST. LOUIS, MISSOURI 63101

IN REPLY REFER TO

SUBJECT: International Airport Dam Mo. ID No. 10661

This report presents the results of field inspection and evaluation of the International Airport Dam. It was prepared under the National Program of Inspection of Non-Federal Dams.

SUBMITTED BY: SIGNED
Chief, Engineering Division

24 AUG 1979
Date

APPROVED BY: SIGNED
Colonel, CE, District Engineer

24 AUG 1979
Date

INTERNATIONAL AIRPORT LAKE DAM
PLATTE COUNTY, MISSOURI

MISSOURI INVENTORY NO. 10661

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY:

BLACK & VEATCH
CONSULTING ENGINEERS
KANSAS CITY, MISSOURI

UNDER DIRECTION OF

ST. LOUIS DISTRICT CORPS OF ENGINEERS

FOR

GOVERNOR OF MISSOURI

APRIL 1979

PHASE I REPORT

NATIONAL DAM SAFETY PROGRAM

Name of Dam	International Airport Lake Dam
State Located	Missouri
County Located	Platte County
Stream	Tributary to Todd Creek
Date of Inspection	17 April 1979

International Airport Lake Dam was inspected by a team of engineers from Black & Veatch, Consulting Engineers for the St. Louis District, Corps of Engineers. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

The guidelines used in the assessment were furnished by the Department of the Army, Office of the Chief of Engineers and developed with the help of several Federal and state agencies, professional engineering organizations, and private engineers. Based on these guidelines, this dam is classified as an intermediate size dam with a high downstream hazard potential. According to the St. Louis District, Corps of Engineers, failure would threaten the life and property of approximately four families and three groups of buildings and would potentially cause appreciable damage to State Highways 92 and O, U.S. Highway 71, Interstate 29, an airport facility road, a sewage treatment plant, and a crossing of one improved road within the estimated damage zone which extends 6.0 miles downstream of the dam.

Our inspection and evaluation indicates the spillway does meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. The spillway will pass the probable maximum flood without overtopping. The spillway design flood recommended by the guidelines is 100 percent of the probable maximum flood. The probable maximum flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

Deficiencies visually observed by the inspection team were erosion, sloughing of the riprap (in the vicinity of spillway downstream), minor settlement of the embankment at the spillway structure, and the presence of trees on the downstream embankment slope. Seepage and stability analyses required by the guidelines were not available.

There were no observed deficiencies or conditions existing at the time of the inspection which indicated an immediate safety hazard. Future corrective action and regular maintenance will be required to correct or control the described deficiencies. In addition, detailed seepage and stability analyses of the existing dam, as required by the guidelines, should be performed. A detailed report discussing each of these deficiencies is attached.

Paul R. Zaman, PE
Illinois 62-29261

Paul B. MacRoberts, PE
Missouri E-15374

Harry L. Callahan, Partner
Black & Veatch

OVERVIEW OF DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
INTERNATIONAL AIRPORT LAKE DAM

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APPENDIX

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the District Engineer of the St. Louis District, Corps of Engineers, directed that a safety inspection of the International Airport Lake Dam be made.

b. Purpose of Inspection. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

c. Evaluation Criteria. Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the Chief of Engineers, in "Recommended Guidelines for Safety Inspection of Dams." These guidelines were developed with the help of several Federal agencies and many state agencies, professional engineering organizations, and private engineers.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances.

(1) The International Airport Dam is an earth structure located in the valley of a tributary to Todd Creek in southeastern Platte County, Missouri (Plate 1). The principal design function is flood control for runoff generated from the airport grounds. The dam acts as the foundation for an airport facility road. This structure has a top width of 48 feet and a length of 1,400 feet. The slopes are protected by a crown vetch vegetal cover and riprap. The emergency spillway is an ogee section located approximately midway across the dam. There is a 24-inch sluice gated pipe running through the spillway which is used to reduce pool elevations below the emergency spillway crest.

(2) A concrete ogee weir spillway with stilling basin was constructed for this dam. The spillway crest is located near the downstream face of the dam, and an approach section with concrete abutment walls is in place. The spillway discharge channel is relatively straight with riprap protection. Trees are evidenced in the lower reach. The channel contains a twin box culvert approximately 250 feet downstream of the stilling basin.

(3) An auxiliary spillway section was also developed for this dam. The main highway entrance leading to the passenger terminals at the airport has been constructed with a low elevation in proximity to the

reservoir pool on the right bank. In the event flood water reaches an elevation of 956 feet m.s.l., the inbound two lane road will overtop. Water will flow around the dam in the grass-lined median strip and enter Todd Creek downstream of the dam.

(4) Four small ponds are located immediately upstream of the reservoir pool and are used for oil skimming. In the event of an aviation fuel spill these ponds will trap the floating fuel/oil before it enters the main reservoir and Todd Creek.

(5) A 24-inch outlet pipe with hand operated sluice gate is located through the ogee weir spillway at Elevation 936.9. The primary function of this outlet is to drawdown the pool below the top of the emergency spillway.

(6) Pertinent physical data are given in paragraph 1.3.

b. Location. The dam is located in southeastern Platte County, Missouri, as indicated on Plate 1. The lake formed by the dam is shown on the United States Geological Survey 7.5 minute series quadrangle map for Ferrelview, Missouri in Section 22 of T52N, R34W.

c. Size Classification. Criteria for determining the size classification of dams and impoundments are presented in the guidelines referenced in paragraph 1.1c above. Based on these criteria, the dam and impoundment are in the intermediate size category.

d. Hazard Classification. The hazard classification assigned by the Corps of Engineers for this dam is as follows: The International Airport Lake Dam has a high hazard potential, meaning that the dam is located where failure may cause loss of life, and serious damage to homes, agricultural, industrial and commercial facilities, and to important public utilities, main highways, or railroads. For the International Airport Lake Dam the flood damage zone extends downstream for 6.0 miles. Within the damage zone are four homes, three groups of buildings, State Highway 92 and O, U.S. Highway 71, and an airport facility road, a sewage treatment plant, and one improved road crossing.

e. Ownership. The dam is owned by the City of Kansas City, Missouri. The Kansas City Department of Aviation, No. 1 International Square, P. O. Box 20047, Kansas City, Missouri 64195, operates and performs maintenance on this structure.

f. Purpose of Dam. The dam forms a 57-acre flood control lake.

g. Design and Construction History. Design data and as-built drawings are available at Burns & McDonnell, Architect-Engineers, the design engineer for this structure. Construction began in 1970 and impoundment of water began in 1971.

h. Normal Operating Procedure. Normal rainfall, runoff, transpiration, and evaporation all combine to maintain a relatively stable water surface elevation. Releases through the 24-inch outlet pipe are made on an as needed basis.

1.3 PERTINENT DATA

a. Drainage Area - 1,440 acres

b. Discharge at Damsite.

(1) Normal discharge at the damsite is through an uncontrolled ogee weir spillway.

(2) Estimated experienced maximum flood at damsite - Unknown.

(3) Estimated ungated spillway capacity at pool elevation 958 (low chord of bridge) 6,800 cfs.

(4) Estimated ungated auxiliary spillway capacity at 958 feet m.s.l. pool elevation approximately 4,000 cfs (elevation at low chord of bridge).

(5) Sluice gate with 24-inch discharge pipe through ogee weir at elevation 936.9 feet m.s.l. (inlet invert).

c. Elevation (Feet Above M.S.L.).

(1) Top of dam - 961.0 ± (see Plate 4)

(2) Spillway crest - 946.0

(3) Streambed at toe of dam - 916.0

(4) Maximum tailwater - Unknown.

d. Reservoir.

(1) Length of maximum pool - 5,800 feet ±

(2) Length of normal pool - 2,700 feet ±

e. Storage (Acre-feet).

(1) At 960 feet m.s.l. - 1,670 (from engineering design data)

(2) Spillway crest - 260 (from engineering design data)

(3) Auxiliary spillway crest - 1,130 (from engineering design data)

f. Reservoir Surface (Acres).

(1) At 960 feet m.s.l. - 149 (from engineering design data)

(2) Spillway crest - 57

g. Dam.

(1) Type - Earth embankment

(2) Length - 1,400 feet

(3) Height - 36 feet ± at spillway

(4) Top width - 48 feet

(5) Side slopes - upstream and downstream faces vary from 1.0 V to 2.5 H to 1.0 V to 1.5 H (see Plate 3)

(6) Zoning - None.

(7) Impervious core - None.

(8) Cutoff - Trench to shale with CL backfill.

(9) Grout curtain - None.

(10) Drainage blanket - Sand - 4 areas connecting to gravel trench and spillway wall weepholes.

(11) Relief wells - 3 (drainage to downstream discharge channel)

h. Diversion and Regulating Tunnel - None.

i. Spillway.

(1) Type - Ogee weir.

(2) Length of spillway - 36.0 feet.

(3) Crest elevation - 946.0 feet m.s.l.

(4) Gates - None.

(5) Upstream channel - Not applicable.

(6) Downstream channel - Open channel comprised of riprap, broken limestone, and shale located near the toe of the downstream embankment slope.

j. Auxiliary Spillway

(1) Type - Broad-crested weir (Roadway)

(2) Length of spillway - 550 feet at elevation 958 feet m.s.l.

(3) Crest elevation - 956.0 feet m.s.l.

(4) Gates - None.

(5) Upstream Channel - Not applicable.

(6) Downstream Channel - Open channel, grass lined.

k. Regulating Outlets - 24-inch sluice gated pipe through spillway.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

Design data were made available by Burns & McDonnell. The data included design and as-built drawings, hydraulic and hydrologic calculations, and boring logs.

2.2 CONSTRUCTION

The dam was constructed in 1970 and 1971. Selected as-built drawings and relevant data were obtained from Burns & McDonnell.

2.3 OPERATION

The maximum recorded loading on the dam is unknown.

2.4 GEOLOGY

The dam is located across a broad, shallow valley formed in loess and glacial till. The overburden consists of the Marshall Silt Loam soil series overlying glacial till. The soil consists of a mixture of sand, silt, clay, and organic matter with silt predominant near the surface and clay predominant at depth. Some alluvial and colluvial soils are present along the stream below the dam. The bedrock of the area is shale of the Lansing and Pedee groups of the Missourian Series, Pennsylvanian System. No outcrops were observed in the area. Subsurface data were taken from the plans of the design engineers. Design drawings indicate the exploration trench was to be excavated to shale between stations 14+00 to 16+00.

2.5 EVALUATION

a. Availability. Engineering data in the form of hydrologic, hydraulic calculations, stability analyses, boring logs, laboratory test data, and appropriate as-built drawings were made available by Burns & McDonnell.

b. Adequacy. The engineering data available were not sufficiently complete to make a detailed assessment of design stability requirements according to the guidelines, and construction or operation of the dam. Seepage analyses necessary to satisfy the requirements of the guidelines were not available.

c. Validity. The engineering data available were not sufficient to determine the validity of the design, construction, and operation.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General. A visual inspection of International Airport Lake Dam was made on 17 April 1979. The inspection team included professional engineers with experience in dam design and construction, hydrology - hydraulic engineering, and geotechnical engineering. Specific observations are discussed below. No observations were made of the condition of the upstream face of the dam below the pool elevation at the time of the inspection.

b. Dam. On both the downstream and upstream slopes erosion has occurred. Trees have been planted on both slopes in proximity to abutments. Some seepage was observed from weepholes located in the spillway side walls. Sloughing and erosion has taken place in an area of about 200 square feet to a depth of from 2 to 3 feet at midslope adjacent to riprap protection on the right side of the spillway. Minor erosion has occurred elsewhere on the dam.

Minor settlement of embankment material in the vicinity of the spillway wall was noted. Both the upstream and downstream embankments have been planted with crown vetch. Minor riprap weathering was noted. Minor animal burrows were located on both slopes. The concrete side walls at the spillway and stilling basin visibly appeared to be in good condition. A relatively large erosion gully was observed on the upstream slope near the right abutment. Surface runoff from the roadway at the dam appears to be the principal cause of erosion gullies.

c. Appurtenant Structures. The inspection team observed the following items pertaining to appurtenant structures. A concrete ogee weir spillway constructed near the middle of the dam appears to be in good condition. The spillway side walls and stilling basin are of concrete construction and also appear to be in good condition. An inspection of the stilling basin below the ponded water level was not performed. There does, however, appear to have been a slight inward displacement of the lower right spillway side wall. The spillway contains a 24-inch discharge pipe with sluice gate. The valve wheel was locked and was not operable although the wheel did turn easily for a short distance. Subsequent to the date of inspection, the 24-inch outlet was observed in the open position and passing flows.

Wing walls and riprap were in place to protect embankment slopes in the vicinity of the ogee spillway. Evidence of some sloughing and subsequent repair was noted on the right side of the spillway.

d. Reservoir Area. No slides or excessive erosion due to wave action were observed along the shore of the reservoir. The shoreline

has been protected by riprap for nearly the entire length. Completion of riprap around the lake is scheduled for this year as reported by an aviation department representative.

e. Downstream Channel. Open channel comprised of broken limestone and shale located near the toe of the downstream embankment slope appears to be in relatively good condition. A small area of erosion on the right bank approximately midway between the spillway and downstream culvert was observed. No remedial measures are warranted.

f. Instrumentation. Three relief wells were observed near the toe of the embankment between the spillway and the right abutment (downstream face). See Plate 3 for location. Relief well RW-1 had an observed water level on the day of inspection of 6 feet below ground level; relief well RW-2 was not measured due to ponded water in the well pit (surface source); and relief well RW-3 had an observed water level on the day of inspection of 3 feet below ground level. From these observations the ground water gradient appears to be toward the discharge channel. The inspection team could not locate the outlet for the relief well manifold pipe.

One of two piezometers shown on available design drawings was found at the dam crest by the inspection team between the spillway and the right abutment. No observation of water levels was possible due to the inability to open the piezometer cap.

Several 4-inch slotted plastic drain pipes were located on the upgrade side of the planted trees. Water was observed to be standing in a number of these pipes. Apparently the plastic pipes are used by the dam owners to water the root zone of the planted trees. There did not appear to be any uniformity in pipe positioning other than being placed on the upgrade side.

3.2 EVALUATION

The various minor deficiencies observed at the time of the inspection are not believed to represent any immediate safety hazard. They do, however, warrant repair and future monitoring and control.

(1) Erosion observed on both slopes of the embankment appears to be the result of surface runoff generated from the paved roadway crossing the dam. Deep erosion gullies have developed near the right abutment and in the vicinity of light standards. It was further observed that minor washouts of the embankment materials have occurred at points along a buried electrical conduit serving the street lights. It is suggested that possible improper compaction and/or the lack of a thick ground cover along the buried conduit was a contributing factor to the erosion.

(2) Trees have been planted in select areas on both slopes of the dam in the abutment areas. Their presence at this time poses no immediate problem, but if the trees are allowed to remain, they may result in future problems. The trees should either be removed or their growth controlled before they become a contributing safety factor.

(3) The riprap protected area to the right of the spillway should be monitored to assure that possible future sloughing is detected as early as possible. There is minor settlement of embankment fill material against the downstream right spillway abutment wall. Although not of immediate concern, this should be continually watched for change.

(4) Riprap weathering, although not considered to be of immediate concern, should be monitored. Riprap should be replaced as necessary to insure that the protected slope is not jeopardized.

(5) Animal burrows if left unchecked can lead to potential problems. Monitoring of the slopes for animal burrows appears to be warranted and measures should be enacted to control wildlife if burrows become widespread.

(6) The slight displacement of the lower right spillway wall should be measured and monitored for future movement. Corrective action should be considered if a professional engineer deems it necessary.

(7) The 4-inch slotted pipes referred to in paragraph 3.1f do not present any immediate concern to the investigating team.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

The pool is primarily controlled by rainfall, runoff, and evaporation. As reported by the owner, releases are made through the 24-inch pipe at the spillway to regulate the reservoir pool to approximately 6 inches below the spillway crest.

4.2 MAINTENANCE OF DAM

According to the owner maintenance is performed as required.

4.3 MAINTENANCE OF OPERATING FACILITIES

According to the owner maintenance is performed as required.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

The inspection team is not aware of any existing warning system for this dam.

4.5 EVALUATION

Discharge releases are made through the 24-inch pipe to regulate the pool to 6 inches below the spillway crest. This provides added insurance of trapping any oil or fuel spills which may occur on the airport property and pass the upstream skimming ponds.

Maintenance at the dam apparently has been performed as evidenced by repair work in the vicinity of the spillway. Additional work should be performed on locating and repairing erosion damage. There is, reportedly, an organized maintenance program at the International Airport which includes the dam and appurtenances.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. A report addressing the 1966 Construction Program, "Final Engineering Report, Drainage Control Facilities" including a discussion of the hydrologic and hydraulic design parameters, methods, and as-built drawings were available for review. Design calculations for hydrology and hydraulics were available.

The embankment and appurtenant structures were designed by Burns & McDonnell. The hydrologic-hydraulic calculations show that inflow hydrographs were computed assuming both a proposed and an ultimate level of development at the International Airport site, as discussed below. Rainfall data for use in the original study were reportedly obtained from the following publications:

1. Technical Paper No. 40: "Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 hours and Return Periods from 1 to 100 Years," U.S. Weather Bureau.

2. Technical Paper No. 25: "Rainfall Intensity -- Duration -- Frequency Curves for Selected Stations in the United States, Alaska, Hawaiian Islands, and Puerto Rico," U.S. Weather Bureau.

3. Hydrometeorological Report No. 33: "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24, and 48 Hours," U.S. Weather Bureau.

Peak runoff rates were computed using the Rational Formula, $Q = CiA$, where:

Q = runoff, acre-feet per hour $\approx$ cfs
C = runoff coefficient
i = intensity of rainfall, inches per hour for a duration equal to time of concentration
A = watershed area, acres.

Runoff coefficients were computed for conditions of ultimate development (weighted for land use factors). The time of concentration was reported to be 40 minutes for conditions of ultimate development considering assumed average critical velocities in the airport storm drainage system.

Peak runoff for the 100-year event was computed to be 3,940 cfs. The controlling hydrologic element of design for the impoundment was the maximum probable storm, 6-hour duration. The design calculation notes

state that the 6-hour event was selected because discharge rates for lesser duration storms can be stored and greater duration storms have discharge rates lesser than the maximum six hour value.

Inflow hydrographs were reported to have been developed by methods discussed in "Hydrology Handbook - ASCE Manual No. 28", "Water Supply & Sewerage" by Steele, and "Sewer Design & Construction - ASCE Manual No. 37."

Burns & McDonnell designed the dam to accommodate a storm producing 23.2 inches of runoff in 10 hours. The maximum water surface elevation obtained was 955.3 feet, m.s.l. or 9.3 feet over the service spillway. Maximum discharge at the spillway was calculated to be 4,500 cfs.

Provisions have been made for flows in excess of 4,500 cfs to spill over the inbound airport entrance road at elevation 956 feet m.s.l. The excess flow is routed around the east end of the dam along the entrance road and enters Todd Creek approximately 900 feet downstream of the dam.

b. Experience Data. The drainage area and lake surface area are developed from USGS Ferrelview, Missouri Quadrangle Map. The spillway and dam layouts are from engineering drawings provided by the design engineer.

c. Visual Observations.

(1) The spillway appears to be in good condition. The discharge channel and stilling basin at the spillway are also in good condition.

(2) Drawdown facilities are available to lower the pool to elevation 936.9 m.s.l. Facilities were observed in operation subsequent to the dam inspection.

(3) A spillway and exit channel are located near the center of the dam. Spillway discharges should not endanger the integrity of the dam due to the fact that overflow from the spillway passes through a concrete stilling basin before entering a riprap lined exit channel.

d. Overtopping Potential. The spillway will pass the probable maximum flood without overtopping the dam. The probable maximum flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The spillway will pass 100 percent of the probable maximum flood without overtopping the dam and therefore, will also pass the 100-year flood, which is smaller than the probable maximum flood. According to the recommended guidelines from the Department of the Army, Office of the Chief of Engineers, a high hazard dam of intermediate size should pass 100 percent of the probable

maximum flood. Failure of the small upstream skimming ponds shown on the 1975 revised USGS map would not have a significant impact on the hydrologic or hydraulic analysis.

According to the St. Louis District, Corps of Engineers, the effect from rupture of the dam could extend approximately 6.0 miles downstream of the dam. There are four dwellings, three groups of buildings, State Highway 92 and O, U.S. Highway 71, Interstate 29, an airport road, sewage treatment plant, and one improved road crossing downstream of the dam which could be severely damaged and lives could be lost should failure of the dam occur.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. Visual observations of conditions which affect the structural stability of this dam are discussed in Section 3, paragraph 3.1b.

b. Design and Construction Data. Design data related to the structural stability of the dam were available from Burns & McDonnell Engineers. Subsurface information, including boring logs and laboratory test data, is provided in "Subsurface Information, MCI Mid-Continent International Airport -- Kansas City, Missouri," Burns & McDonnell Engineers, Kivett & Myers Architects, Job No. 65-32D1. Dam stability analyses in the form of computer input and print-out sheets were provided by Burns & McDonnell Engineers. Other information included Mohr-circle plots of several triaxial compression test results, calculations to establish the position of the seepage line to the drainage blanket, and stability calculations of the foundation for the spillway wall.

(1) Stability analyses of the upstream and downstream slopes for end-of-construction and steady-seepage conditions were performed during embankment design.

The stability analyses for steady-seepage conditions were performed by the modified Swedish Slip Circle method of analysis following the procedure set forth in the Corps of Engineers Civil Works, Engineering Manual, EM 1110-2-1902. Shear strength parameters were selected from the results of triaxial compression tests (consolidated, undrained, or C-U) on remolded samples of representative fill material and triaxial compression tests and unconfined compression tests on undisturbed samples of the embankment foundation materials.

Stability analyses for the steady-seepage condition were performed on the upstream slope for the maximum height section and on the downstream slope for the slope section adjacent to the spillway. The typical embankment slopes are 2.5 H to 1.0 V except in the area adjacent to the spillway where the slope is 1.5 H to 1.0 V for the upper 8 feet of the upstream slope and 24 feet on the downstream slope.

The shear strength parameters used for the steady-seepage stability analyses of the upstream slope were c = 500 psf and = 14 degrees for the embankment material and c = 750 psf and = 14 degrees for the foundation. The reservoir level was input at elevation 946 which is the spillway crest elevation. The minimum calculated factor of safety was 2.19.

Two sets of shear strength parameters were used by the designer in the stability analyses of the downstream slope for steady seepage loading. One analysis was performed using c = 500 psf and ϕ = 14 degrees for the embankment material and c = 750 psf and ϕ = 14 degrees for the foundation. The position of the seepage line used in this analysis was controlled by the 2.0 feet thick horizontal sand blanket extending 20 feet laterally into the embankment from each spillway wall. The minimum factor of safety was computed to be 1.35.

The second analysis of the downstream slope for the steady seepage condition was performed using c = 250 psf and ϕ = 17 degrees for the embankment material while the cohesion and friction angle for the foundation material remained unchanged. The minimum factor of safety was computed to be 1.09.

(2) Seepage analyses for the embankment were not available. Three relief wells were installed about 120 feet downstream of the dam centerline. The spacing between relief well RW-1 and relief well RW-2 is 90 feet; spacing between relief well RW-2 and relief well RW-3 is 100 feet. The relief wells penetrate the overburden, and the slotted pipe sections are installed in a sandy clay and sand layer immediately above the shale foundation.

(3) Available stability analyses are included for the end-of-construction and steady seepage (Case III) loading conditions. The end-of-construction loading condition is not significant to the inspection program.

Loading conditions (Case I, II, and IV) consistent with Chapter 4, Table 4 of the guidelines referenced in paragraph 1.1 c above were not included in the stability analyses.

Loading conditions for Case I and II are considered not appropriate for this dam. The reservoir pool is controlled by the fixed crest overflow spillway. The time the embankment is exposed to high water above the crest is considered short-term even for the maximum inflow and discharge conditions. The depth of saturation into the clay-type embankment would be shallow and therefore sudden drawdown would have no significant impact on the slope stability. The reservoir level is maintained essentially constant by the fixed crest overflow at elevation 946 except during short periods of high runoff or drought. Intermediate reservoir stages or variations in the pool elevation are not expected to occur.

The factors of safety calculated by the designer for the steady seepage condition using two sets of strength parameters were less than the factor of safety of 1.5 suggested in the guidelines:

Embankment Strength Parameters	Factor of Safety
C = 500 psf, ϕ = 14°	1.35
C = 250 psf, ϕ = 17°	1.09

The decrease in the selected value of cohesion from 500 psf to 250 psf significantly lowered the calculated factor of safety even though the friction angle increased three degrees. Our stability evaluation of the design slope indicated that a factor of safety of 1.5 (as suggested in the guidelines for the steady-seepage loading condition) could be obtained using an assumed cohesion equal to 400 psf and a friction angle of 17 degrees. A shear strength of 400 psf cohesion from a triaxial consolidated, undrained test for typical embankment material existing at this site remolded to 95 percent of Standard Proctor maximum density does not seem unrealistic. It is our opinion that a shear strength value higher than 250 psf could be justified with a larger number of tests.

Observation made during the inspection indicates that the phreatic surface is below the ground surface at the downstream toe of the dam and across the deep sediment portion of the valley. Water levels were observed in the relief wells. The header pipe which connects the wells and discharges into the spillway discharge channel according to the design drawings could not be found.

c. Operating Records. Operational records were not available.

d. Post Construction Changes. No known post construction changes.

e. Seismic Stability. The dam is located in Seismic Zone 1 which is a zone of minor seismic risk. A properly designed and constructed earth dam using sound engineering principles and conservatism should pose no serious stability problems during earthquakes in this zone.

The seismic stability of an earth dam is dependent upon a number of factors: The important factors being embankment and foundation material classification and shear strengths; abutment materials, conditions, and strength; embankment zoning; and embankment geometry. Static stability analyses to assess the seismic stability of this embankment were not available. An assessment of the seismic stability should be included as part of the stability analysis required by the guidelines. It is anticipated that no serious stability problems would be experienced at this dam during an earthquake characteristic of Seismic Zone 1.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. Several items noted during the visual inspection by the inspection team which should be monitored or controlled are erosion of the upstream embankment slope, erosion of the downstream slope, sloughing and settlement near the spillway (right side) and a controlled stand of trees on both the downstream and upstream embankment slopes.

b. Adequacy of Information. The inspection team considers the available hydrologic-hydraulic and pertinent physical data, in addition to the available stability analysis, used in conjunction with the observed visual conditions are sufficient to support the conclusions herein. However, seepage and stability analyses comparable to those required in the guidelines are necessary to satisfy the requirements of the guidelines.

c. Urgency. It is the opinion of the inspection team that a program should be developed within the next year to implement remedial measures recommended in paragraph 7.2b. If the safety deficiencies listed in paragraph 7.1a are not corrected, they will continue to deteriorate and lead to a potential of failure.

d. Necessity for Phase II. The Phase I investigation does not raise any serious questions relating to the safety of the dam or identify any serious dangers that would require a Phase II investigation.

e. Seismic Stability. This dam is located in Seismic Zone 1. Static stability analyses to assess the seismic stability of this embankment was not available. Seismic stability analyses are needed to satisfy the guideline requirements.

7.2 REMEDIAL MEASURES

a. Alternatives. Deficiencies observed at the time of inspection can be remedied through normal maintenance practices under the supervision of an engineer experienced in the design, construction, or maintenance of earth structures.

b. O&M Maintenance and Procedures. The following O&M maintenance and procedures should be implemented to correct the deficiencies observed at the time of inspection. Although these are considered to be of minor magnitude at this time, if left unattended or unrepaired each could ultimately become a potential source of failure.

(1) Fill and compact erosion gullies to original specifications. Provide slope protection either through the use of vegetal ground cover

or riprap. It is suggested that a curb, gutter, and inlet system be designed and constructed for the roadway crossing the structure.

(2) Monitor and replace riprap as considered necessary.

(3) Analyze extent of tree growth since planting. An engineer experienced in the maintenance of dams should be consulted to recommend procedures to either control the growth or remove the trees.

(4) A detailed inspection of the dam and appurtenances should be made at least every year by an engineer experienced in design and construction of dams. More frequent inspections may be required if additional deficiencies are observed or the severity of the reported deficiencies increases.

(5) Seepage and stability analysis should be performed by a professional engineer experienced in the design and construction of dams.

(6) Analysis of the sloughing and settlement problems on the downstream slope near the spillway (right side) should be performed by a qualified engineer.

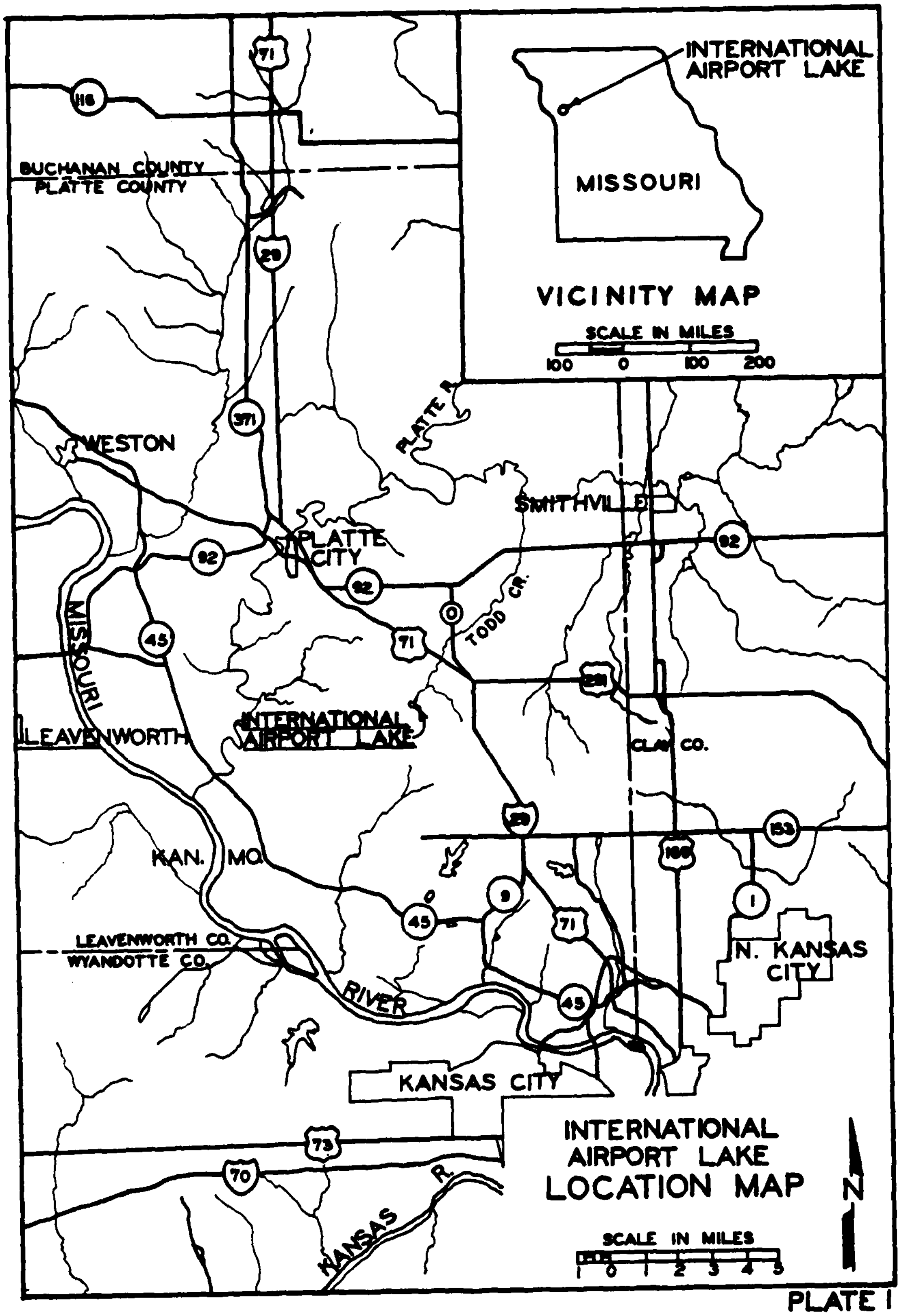
INTERNATIONAL AIRPORT LAKE
MISSOURI
VICINITY MAP
SCALE IN MILES
100 0 100 200
BUCHANAN COUNTY
PLATTE COUNTY
WESTON
PLATTE CITY
PLATTE R.
SMITHVILLE
TODD CR.
MISSOURI
LEAVENWORTH
INTERNATIONAL AIRPORT LAKE
CLAY CO.
KAN. MO.
LEAVENWORTH CO.
WYANDOTTE CO.
RIVER
N. KANSAS CITY
KANSAS CITY
KANSAS R.
INTERNATIONAL AIRPORT LAKE LOCATION MAP
SCALE IN MILES
1 0 1 2 3 4 5
N

PLATE I

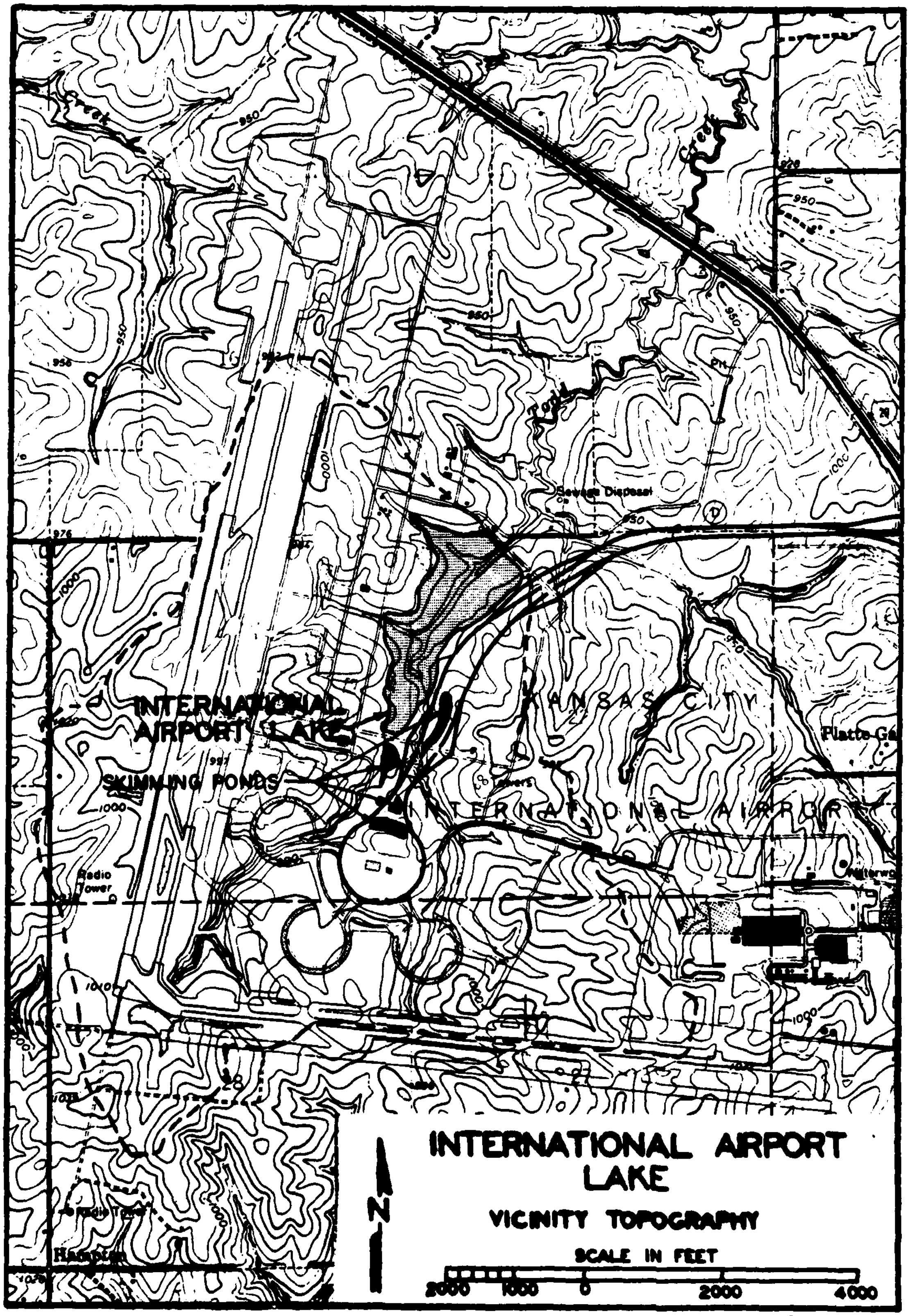
INTERNATIONAL AIRPORT LAKE
SKIMMING PONDS
Sewage Disposal
KANSAS CITY
Platte
Radio Tower
950
1000
INTERNATIONAL AIRPORT
LAKE
VICINITY TOPOGRAPHY
SCALE IN FEET
2000
1000
0
2000
4000
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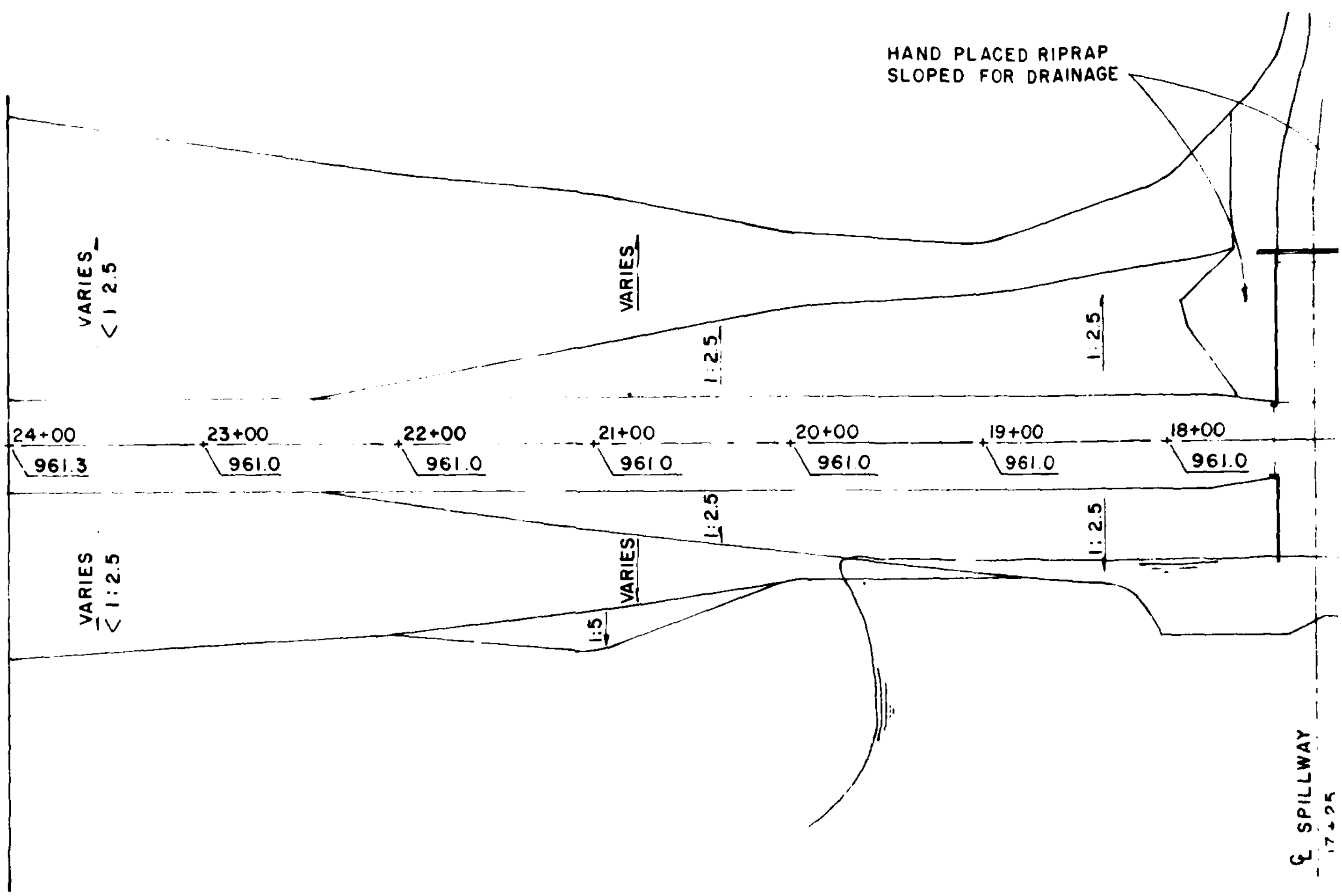
PLATE 2



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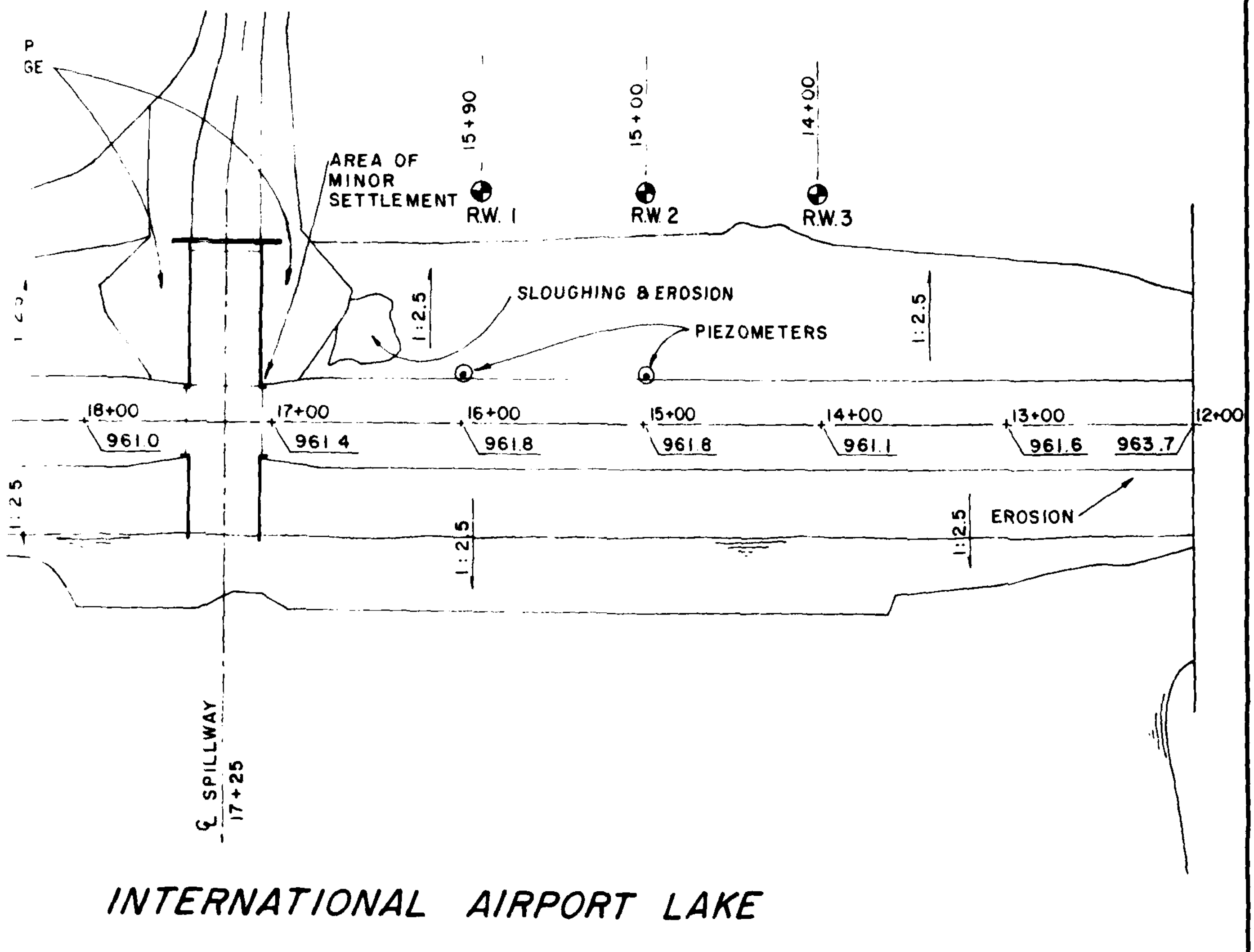
NOTE: PLAN INFORMATION OBTAI
"CONSTRUCTION RECORD"

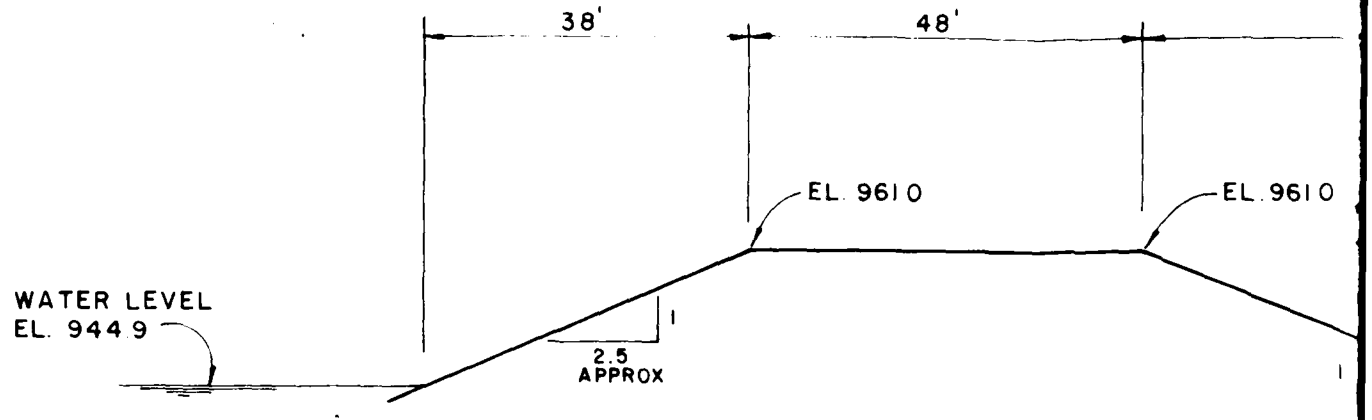


NFORMATION OBTAINED FROM
RUCTION RECORD" DRAWINGS.

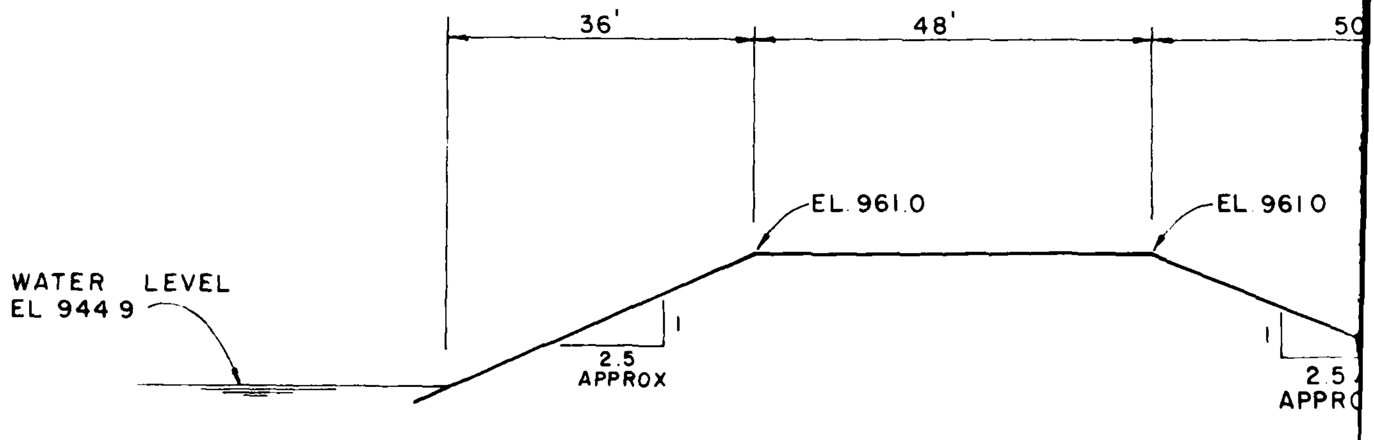
2

INTERNATIONAL AIRPORT LAKE
PLAN

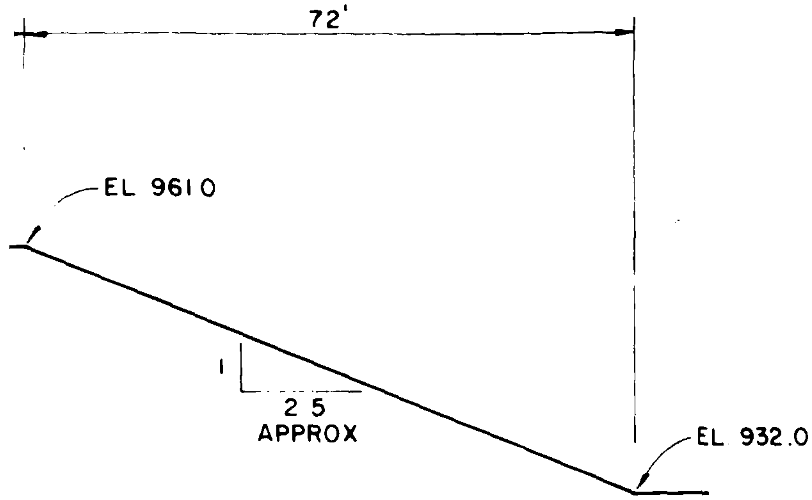
PLATE 3

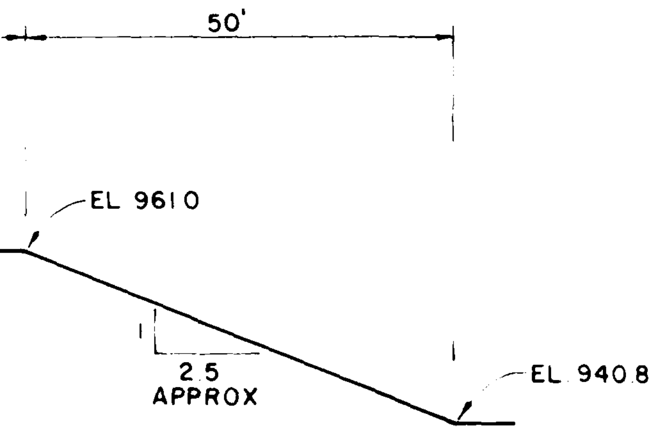


SECTION TAKEN AT APPROX. 16+00



SECTION TAKEN AT APPROX. 19+00

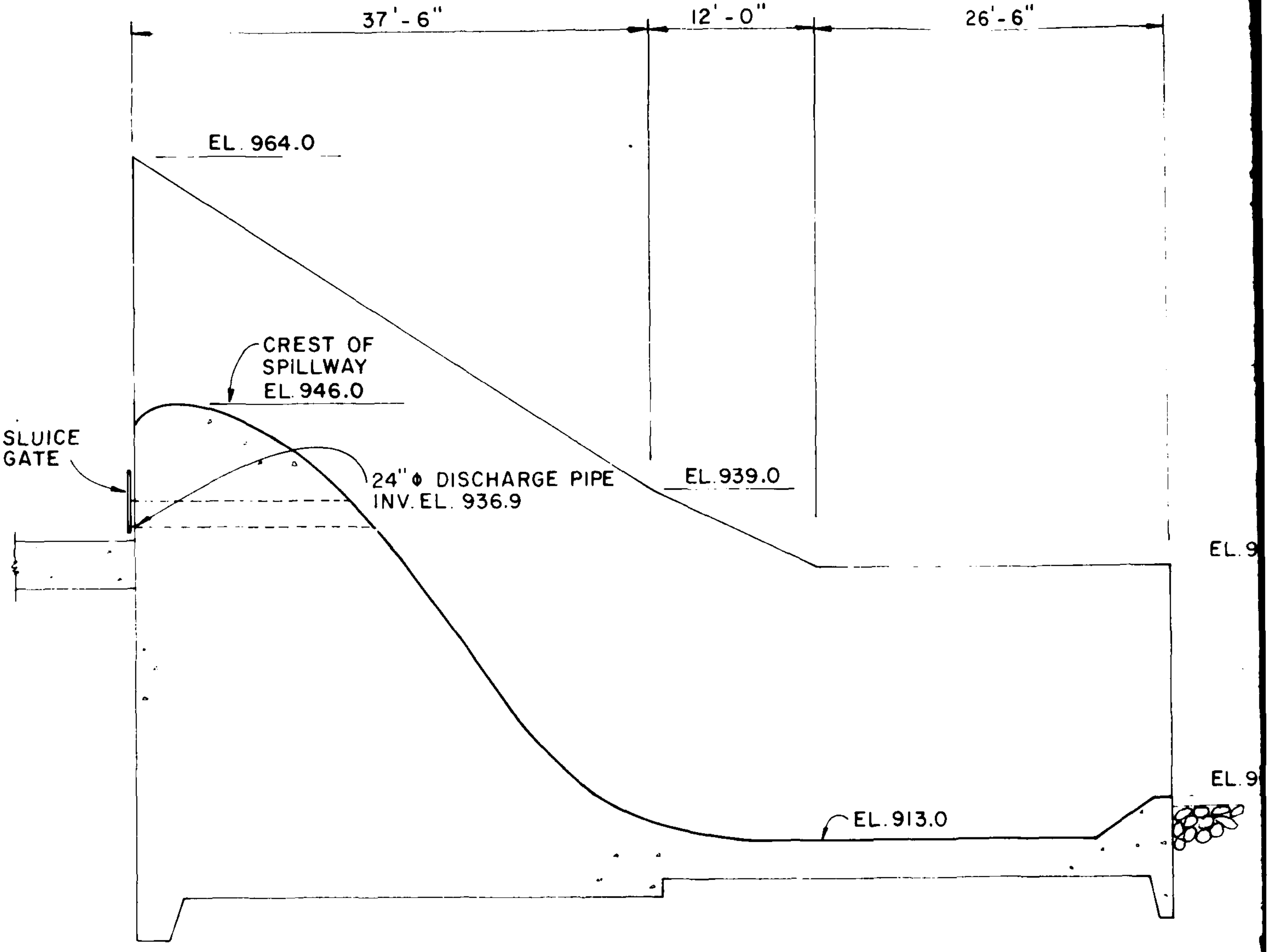




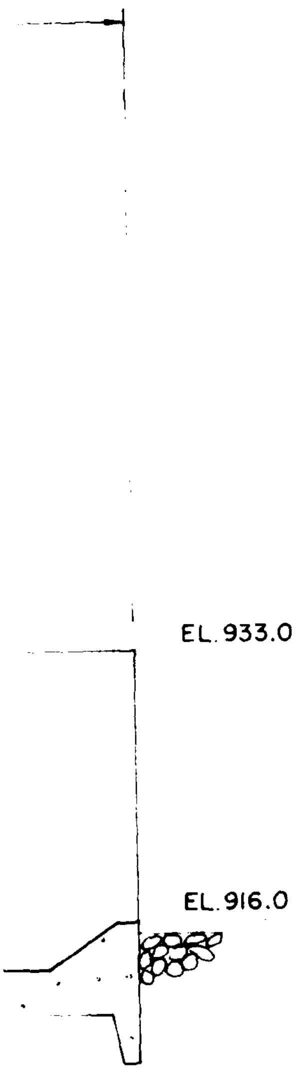
2

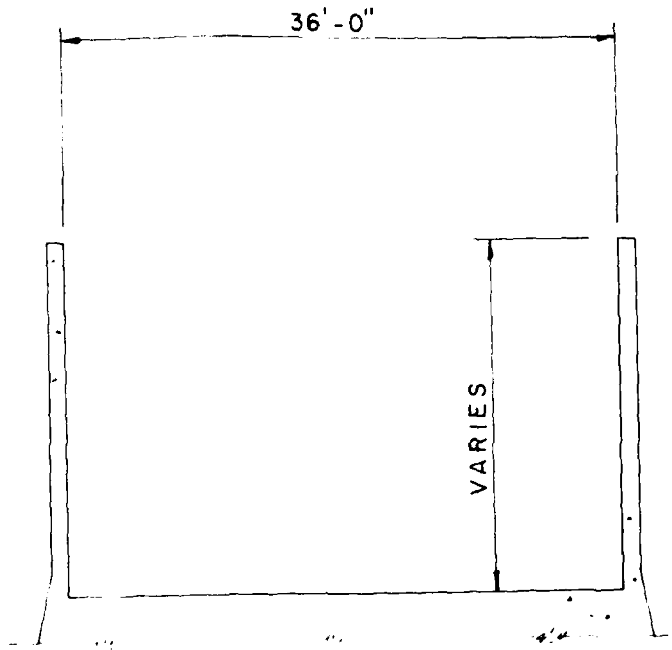
INTERNATIONAL AIRPORT LAKE
TYPICAL SECTIONS

PLATE 4



SPILLWAY LONGITUDINAL SECTION





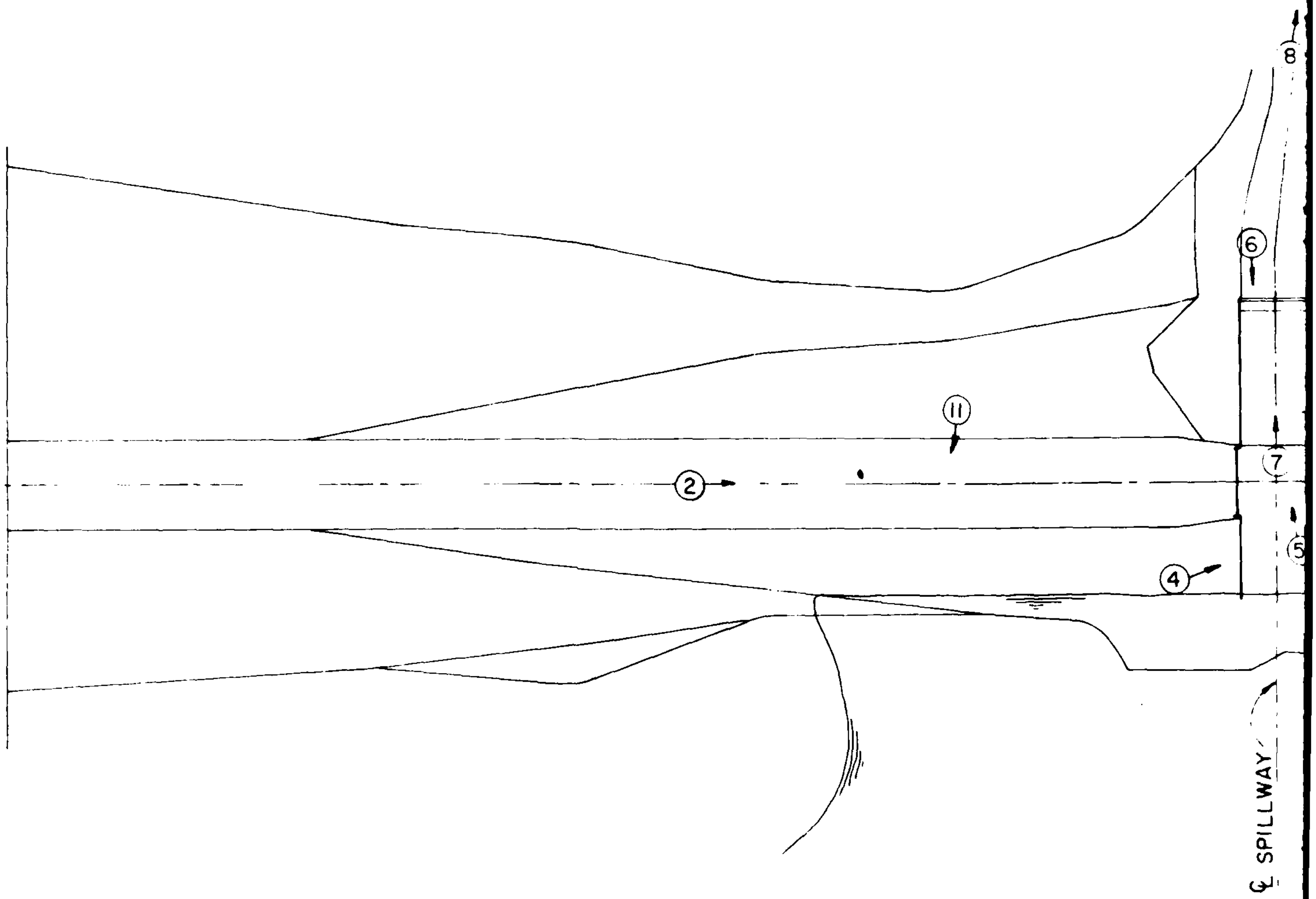
SPILLWAY CROSS SECTION

NOTE: SPILLWAY INFORMATION OBTAINED FROM "CONSTRUCTION RECORD" DRAWINGS.

2

INTERNATIONAL AIRPORT LAKE
SPILLWAY SECTIONS

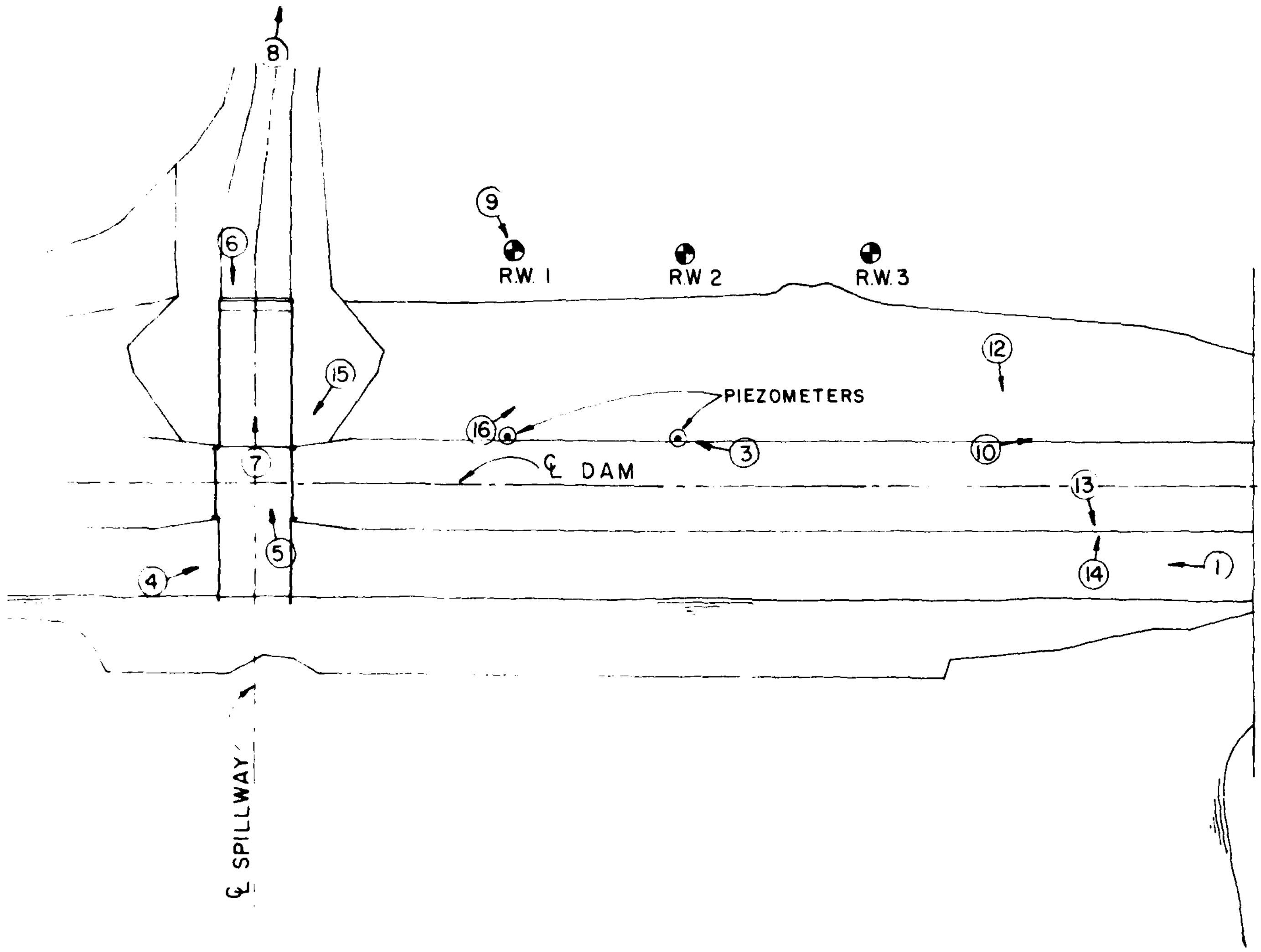
PLATE 5



INTERN

LEGEND

(1) PHOTO LOCATION AND DIRECTION



INTERNATIONAL AIRPORT LAKE

N

AND

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INTERNATIONAL AIRPORT LAKE
PHOTO INDEX

PLATE 6

PHOTO 1: UPSTREAM FACE OF DAM

PHOTO 2: CREST OF DAM

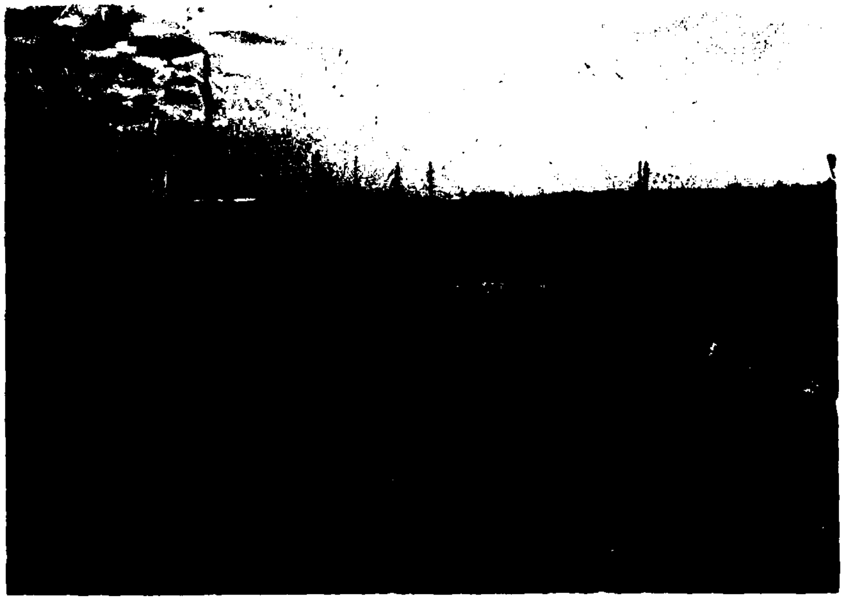

PHOTO 3: DOWNSTREAM FACE OF DAM

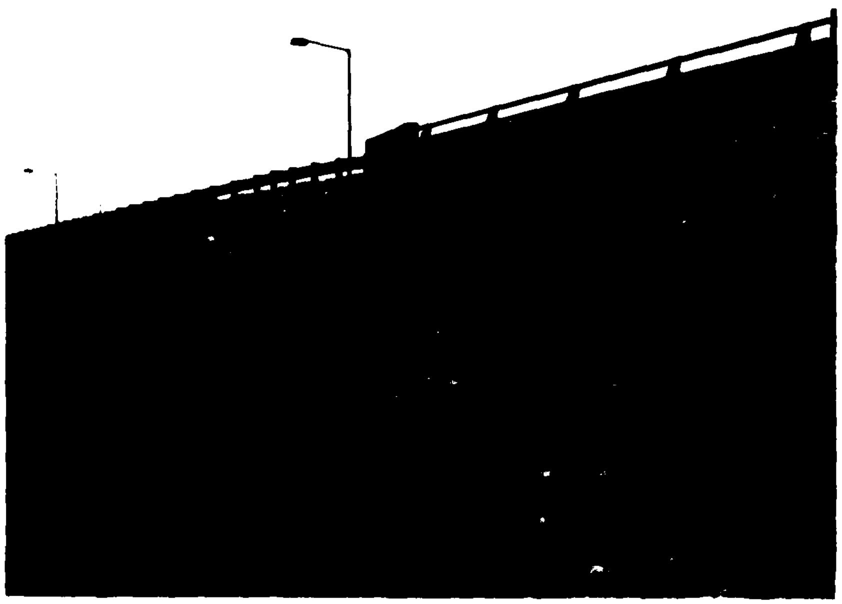

PHOTO 4: SPILLWAY APPROACH

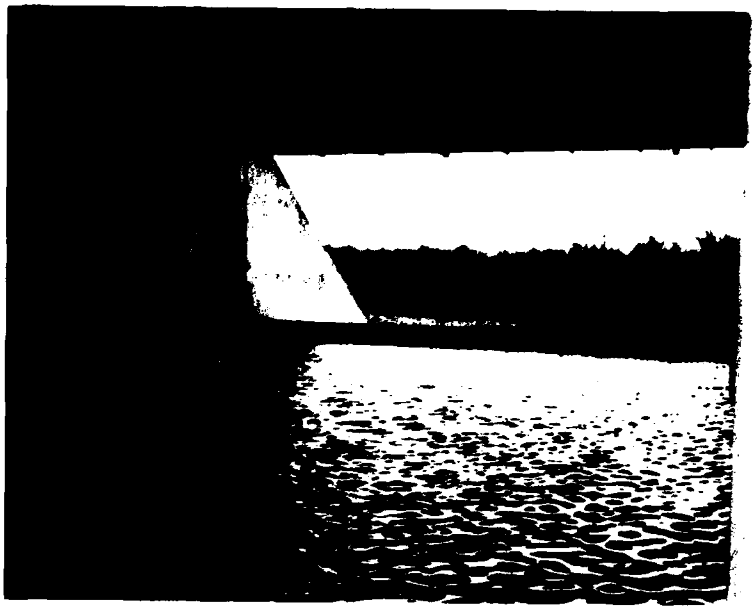

PHOTO 5: SPILLWAY CREST LOOKING DOWNSTREAM

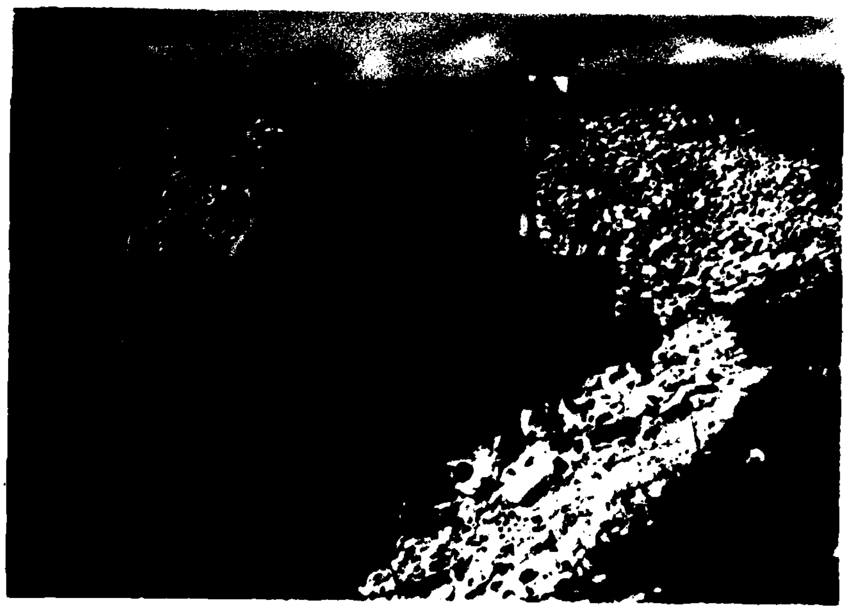

PHOTO 6: SPILLWAY LOOKING UPSTREAM

PHOTO 7: SPILLWAY DISCHARGE CHANNEL

PHOTO 8: CHANNEL BELOW SPILLWAY

PHOTO 9: RELIEF WELL NO. 1 AT TOE OF DAM

PHOTO 10: EROSION OF EMBANKMENT AT CREST OF DOWNSTREAM SLOPE

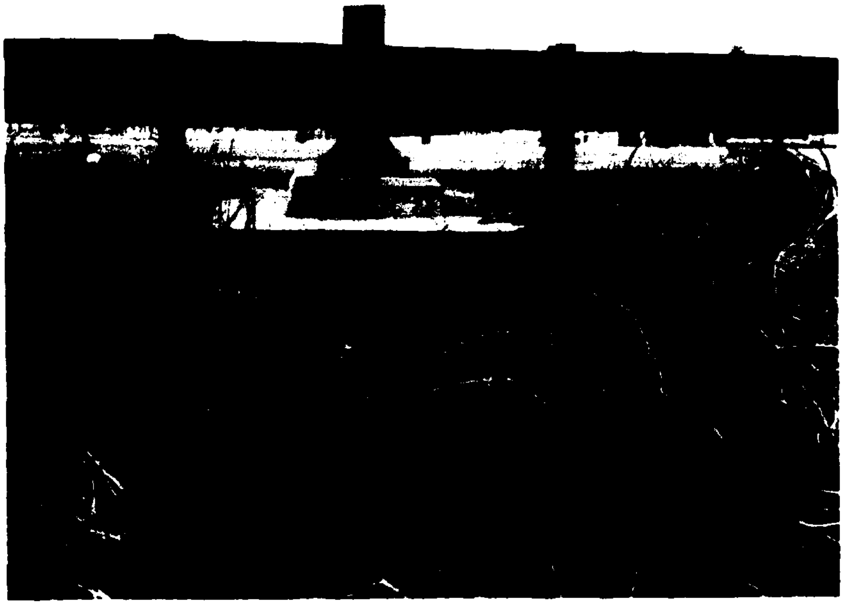

PHOTO 11: UNDERMINING OF LIGHT STANDARD AT CREST OF DOWNSTREAM SLOPE

PHOTO 12: EROSION AND PLANTED TREES DOWNSTREAM SLOPE

PHOTO 13: EROSION AT CREST OF UPSTREAM SLOPE

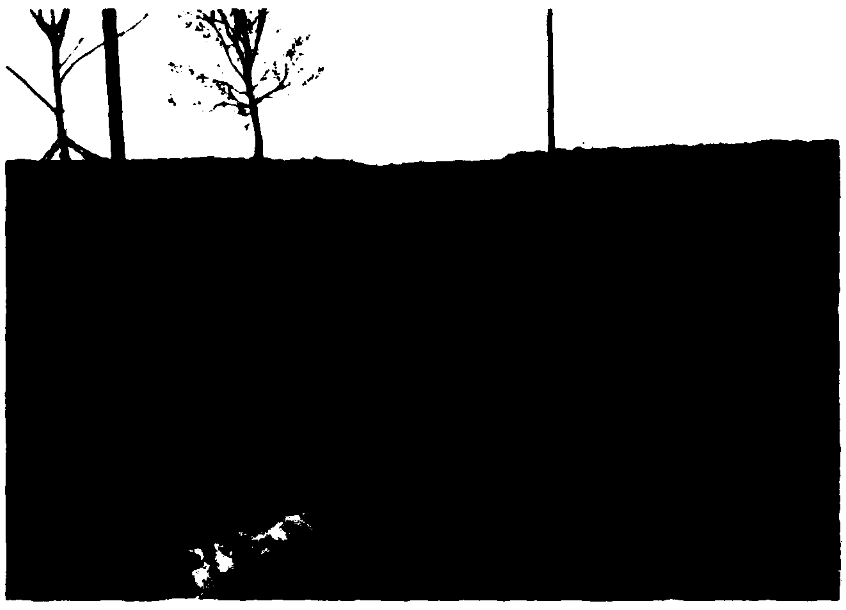

PHOTO 14: EROSION OF UPSTREAM SLOPE

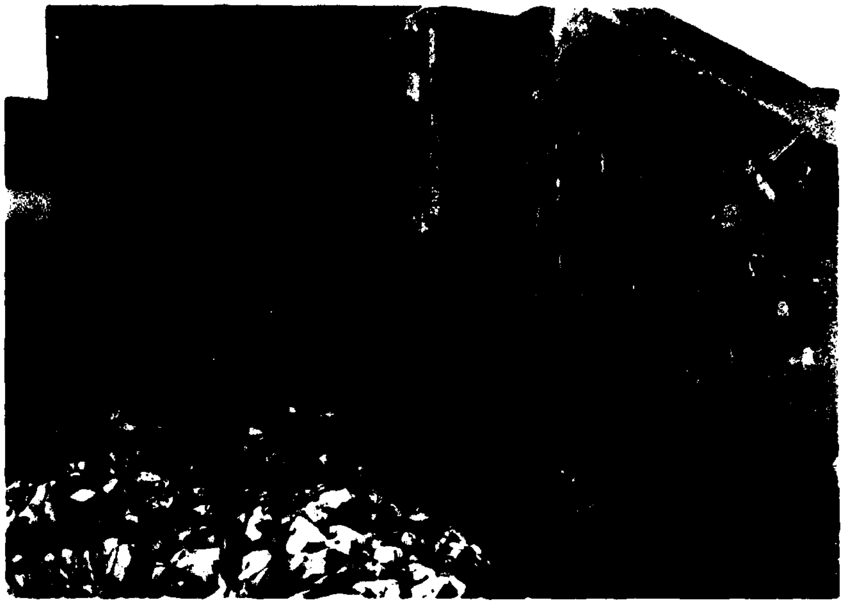

PHOTO 15: SETTLEMENT OF EMBANKMENT AT SPILLWAY STRUCTURE

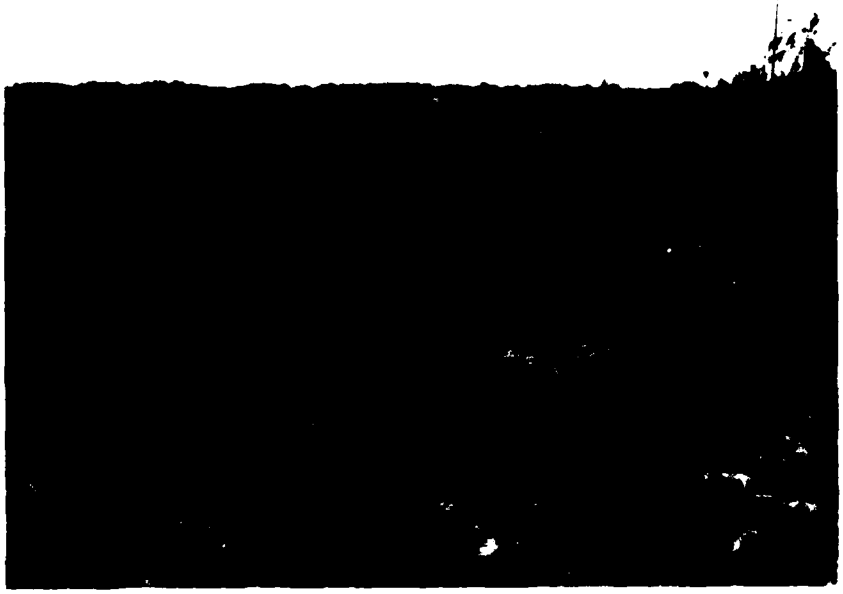

PHOTO 16: SLOUGHING OF EMBANKMENT MATERIAL DOWNSTREAM SLOPE

APPENDIX A

HYDROLOGIC COMPUTATIONS

HYDROLOGIC COMPUTATIONS

1. The Soil Conservation Service (SCS) dimensionless unit hydrograph and HEC-1 (1) were used to develop the inflow hydrographs and hydrologic inputs are as follows:

a. Twenty-four hour, probable maximum precipitation determined from U.S. Weather Bureau Hydrometeorological Report No. 33.

200 square mile, 24 hour rainfall inches	- 24.5
10 square mile, 6 hour percent of 24 hour 200 square mile rainfall	- 101%
10 square mile, 12 hour percent of 24 hour 200 square mile rainfall	- 120%
10 square mile, 24 hour percent of 24 hour 200 square mile, rainfall	- 130%

b. Drainage area = 1,440 acres.

c. Time of concentration: $Tc = (11.9 \times L^3/H)^{0.385} = 0.67$ hours = 40 minutes (L = length of longest watercourse in miles, H = elevation difference in feet) (2)

d. Losses were determined in accordance with SCS methods for determining runoff using a curve number of 86 and antecedent moisture condition III. The hydrologic soil groups in the basin where B, C, and D.

2. Spillway release rates are based on the weir equation.

Weir equation:

$Q = CLH^{1.5}$ (C = 4.0, L = 36.0 feet, H is the head on weir).

Discharge rates over the adjacent road are also based on the weir equation:

$Q = CLH^{1.5}$ (C = 2.63, L = 80 to 550 feet).

3. The elevation-storage relationship above normal pool elevation was obtained from the available design calculations.

4. Floods are routed through the spillway using HEC-1, modified Puls to determine the capability of the spillway.

(1) U.S. Army Corps of Engineers, Hydrologic Engineering Center, Flood Hydrograph Package (HEC-1), Dam Safety Version, July 1978, Davis, California.

(2) U.S. Department of the Interior, Bureau of Reclamation, Design of Small Dams, 1974, Washington, D.C.

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********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  25 SEP 78
********************************
     1            A1MISSOURI DAM INSPECTION PROGRAM
     2            A2ST. LOUIS DISTRICT US ARMY CORPS OF ENGR
     3            A3KCI LAKE NO.10661
     4            B    288       0       5       0       0       0       0       0       0       0
     5            B1     5
     6            J      1       2       1
     7            J1   0.5     1.0
     8            K      0       1                               3
     9            K1   24 HR PMF INFLOW
    10            M      1       2    2.25                       1
    11            P           24.5     101     120     130
    12            T                                                     -1     -91
    13            W2               .40
    14            X                        1
    15            K      1       2                                       1
    16            K1   ROUTE THROUGH SPILLWAY
    17            Y                                1       1
    18            Y1     1                                              -1      -1
    19            Y4  946.    947.    948.    949.    950.    951.    952.    953.    954.    955.
    20            Y4  956.    957.    958.
    21            Y5C.        125.    303.    700.   1125.   1600.   2175.   2800.   3500.4250.
    22            Y5 5075.  6795.  10895.
    23            SS     0    190.    350.1130.   3210.
    24            SE946.    949.    951.    958.  970.
    25            S$  946.
    26            SD  961.
    27            K     99
```

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
949.7	949.6	949.6	949.5	949.5	949.4	949.4	949.3	949.3	949.3
949.2	949.2	949.1	949.1	949.1	949.0	949.0	949.0		

PEAK OU[illegible]W IS 11284. AT TIME 16.33 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	11284.	5095.	1753.	1753.	506920.
CMS	320.	144.	50.	50.	14298.
INCHES		21.07	28.99	28.99	28.99
MM		535.05	736.43	736.43	736.43
AC-FT		2527.	3477.	3477.	3477.
THOUS CU M		3116.	4289.	4289.	4289.

********** ********** ********** ********** **********

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS: RATIO 1 .50	RATIO 2 1.00
HYDROGRAPH AT	1	2.25	1	8791.	17582.
	(	5.83)	(	248.94)(	497.88)(
ROUTED TO	2	2.25	1	3596.	11284.
	(	5.83)	(	101.81)(	319.53)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	946.00	946.00	961.00
STORAGE	0.	0.	1650.
OUTFLOW	0.	0.	13619.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.50	954.13	0.00	698.	3596.	0.00	16.58	0.00
1.00	958.13	0.00	1153.	11284.	0.00	16.33	0.00

END

DATE
FILMED
12-81

DTIC